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Bearing Capacity of Foundations subjected to Impact Loads

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1 Introduction

In the design process for foundations, the bearing capacity calculations are normally restricted to monotonic loads. Even in cases where the impact load is of significance the dynamic aspects are neglected by use of a traditional deterministic ultimate limit state analysis. Nevertheless it is common knowledge that the soil under such circumstances normally will produce a reaction, which exceed the static capacity. A new model, which takes these significant properties into account is presented.

To utilize this dynamic property it is sometimes assumed that the foundation is resting on an elastic halfspace, meaning a homogeneous, isotropic, elastic semi-infinite body of soil (Das 1983 and Kortenhaus et al. 1993). Such an assumption seems reasonable in the serviceability limit state where only small strains occur and the methods might under these conditions produce reliable results.

Generally the impact load under consideration is of such a magnitude that the monotonic design load or even the static bearing capacity might be exceeded. Due to this fact and a pronounced plastic behaviour of the soil, it is clear that the mentioned models can not describe the motion of the foundation near failure. The establishment of an alternative model is therefore needed.

The above mentioned facts lead to the first alternative model based on perfect plasticity, which implies a qualitative evaluation of different failure modes. Since the problems usually appears in design situations involving structures of a great size (vertical breakwaters, gravitational platforms and bridge piers) the problem are reduced to include plane failure modes. Theoretically these modes must be statically as well as the kinematically admissible, but since it is difficult to fulfil both conditions it is chosen to use the upper bound theory, which means that the failure modes are at least presumed to be kinematic admissible. The dynamic aspects are introduced as the geometrical change of the rupture figure, the inertia of the accelerated mass and an eventual increase in the soil strength. These constituent parts collectively form the equation of motion, for which reason the dynamic bearing capacity must be determined at an admissible irreversible displacement.

2 Description of the Dynamic Model

In a traditional upper bound solution the bearing capacity emanate from the strength parametres, the specific gravity of the soil and any overburden pressure. However the influence from the different elements will depend on whether drained or undrained conditions are considered. It is in the following chosen only to consider undrained conditions, but the approach to drained conditions is straight forward.
Since the model is based on perfect plasticity the foundation will stay at rest until the static load carrying capacity is exceeded. An exceedance will then cause the solid bodies of soil and the superstructure to move along the failure lines prescribed by the displacement diagram, and since the material is plastic any displacement is permanent and the movements will stop whenever the velocity of the bodies equals zero. Subsequently the dynamic elements are introduced by means of the acceleration in the displacement diagram, for which reason it is of great importance that the applied failure mode produces reliable results. Thus the dynamic load carrying capacity will depend on the choosen failure mode and hence it is found convenient to express the dynamic capacity by an overloading factor which solely describes the permissible overload in proportion to the static load carrying capacity of the current failure mode.

A more detailed description of the dynamic elements is given below.

2.1 The Dynamic Strength of Soil

As mentioned before it is found that the soil due to dynamic loading might produce an additional strength. This phenomenon has especially been studied by undrained triaxial compression tests with varying strain rates. The tests have been performed on cohesive as well as non-cohesive soils with quit different results.

Cohesive soils are found to exhibit a change in undrained shear strength (c_u) as a function of the strain rate ($\dot{\varepsilon}$) during loading. This partially viscous behavior is illustrated by Bjerrum 1971 and by Kulhawy et al. 1990. Thus the relationsship between undrained shear strength of clay and the strain rate may be described as.

$$c_{ud} = c_u + \Delta c_u^d \log \dot{\varepsilon} \approx c_u + \Delta c_u^d \dot{\varepsilon} \qquad (1)$$

The latter part is convenient, but only valid for small variations in the strain rate.

A similar relationsship is not observed for sand, where the strength seems independent of the strain rate. However, it is found that the failure under undrained conditions is controlled by the same limiting conditions as found for drained failure (Ibsen 1995). This means that the common stress path that in the literature often is considered to be the undrained failure envelope does not present any failure state in the sand; a shear strength corresponding to the drained strength must be used.

2.2 Forces of Inertia

In order to calculate the dynamic load carrying capacity it is necessary to include the inertia of the soil and the superstructure. The mass of the superstructure is often considerable and can therefore not be neglected.

By presuming that the soil and the structure moves as rigid bodies the displacement diagram can be used for the determination of the systems inertia. Thus the displacement diagram prescribes the direction and the magnitude of the displacement and the equivalent acceleration of the bodies.

2.3 Geometrical Changes

The development of failure in the foundations might in general cause significance damage to the superstructure. However, it is found that the development of such failure often results in geometrical changes which improve the overall stability. Whenever the bearing capacity is execeeded, soil will be pushed up by the side of the foundation and act as an overburden pressure. The upheaval in front of the foundation results in an increase in the stabilizing earth pressure, whereas the subsidence behind the foundation reduces the propulsive earth pressure. Coincident with the change in earth pressure, the displacement of the rigid bodies results in a shortening of the failure lines and the bearing capacity is hereby reduced. Totally the geometrical changes will either lead to a strengthening of the foundation or precipitate failure, for which reason the effect must be evaluated in each case.

2.4 Simplified Representation of the Load

In this context the load is presumed to consist in an external action, which within a short time exceeds the static load carrying capacity. Since various elements in general are important for the appearance of the actual load history the history are often jagged and inapplicable for mathematical formulations, for this reason a simplified representation of the load is necessary. The choice of such a simplified load substitution is found to be of great significance for the irreversible displacement and hereby for the bearing capacity. It is however desirable to attain a formulation of the impact load which in a simple way takes the significant properties into account.

To illustrate the characteristics of an impact load, a typical record of a maximum horizontal wave load is shown in Figure 2.1.
For this type of loads some interesting properties can be observed (Oumeraci et al. 1992).

- The impact has a magnitude (F_{dyn}) that is about twice the quasi-static load (F_{static}).
- The quasi-static load is only exceeded for a short period of time.

For the description of this phenomenon Oumeraci et al. 1992 have proposed a so-called

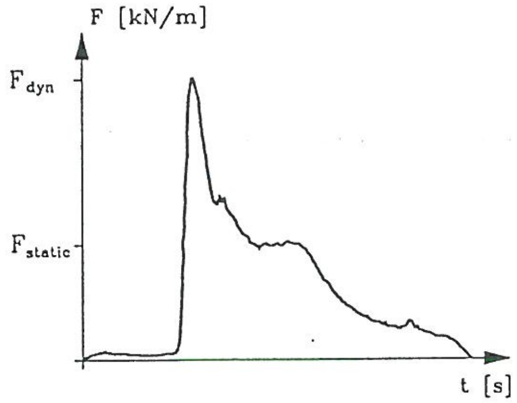

Fig. 2.1 Sketch of a typical horizontal wave load

"church-roof" load history (cf. Figure 2.2). Alternatively, a triangular load history is proposed whenever F_{dyn} is a factor two or more greater than F_{static}. The influence of the quasi-static part of the load is then neglected and the load is solely described by the maximum load, the rise time (t_r) and the decay time (t_d).

In cases where the rise time is very short, it is found convenient to use a simple exponential

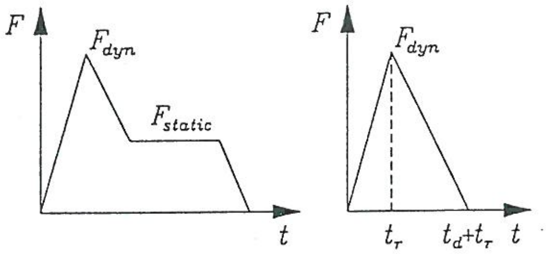

Fig. 2.2 Simple load substitutions. Redrawn from Oumeraci et al. 1992.

function for the description of the load. In this way a continuous formulation which more appropriately describes the decay and contain the total amount of energy in the the load signal is obtained. The load history is given by the maximum load and a constant describing the decay.

In both cases the maximum load is related to the static load carrying capacity of the current failure mode (q_u) by an overloading factor (S). The load substitutions are given in the equation 2 and 3, corresponding to the triangular and exponential load substitution, respectively. The load substitutions are furthermore illustrated in Figure 2.3.

$$q_d = \begin{cases} q_u\left(1-\frac{1-S}{t_a}t\right) & 0 \le t \le t_a \\ Sq_u\left(1-\frac{t-t_a}{t_b-t_a}\right) & t_a \le t \le t_b \\ 0 & t \ge t_b \end{cases} \qquad (2)$$

$$q_d = Sq_u e^{-kt} \qquad (3)$$

It is noticed, that in case of a triangular load history the static load carrying capacity has to be exceeded before the soil and the superstructure are affected.

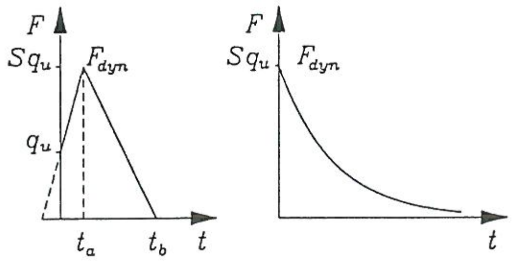

Fig. 2.3 Applied load substitutions.

2.5 Equation of Motion

It is in the following implied that the response can be described by uncoupled motions. Assuming that the time varying force acts in the same direction as the decisive displacement, it becomes possible to describe the motion by a system with only one degree of freedom. The equation of motion is established by demanding a balance between internal and external forces.

$$m\ddot{\alpha} + \Delta q_u^d \dot{\alpha} + \Delta q_u \alpha = f(t) \qquad (4)$$

where

Δq_u^d : dynamic strengthning of the soil.
Δq_u : effect of the geometrical changes.
m : total mass of the system.
$f(t)$: external force.
α : displacement.

The formulation is valid as long as the displacement is increasing.

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